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BENDING STRENGTH OF LARGE ALASKAN SITKA SPRUCE AND WESTERN HEML--ETC(U)
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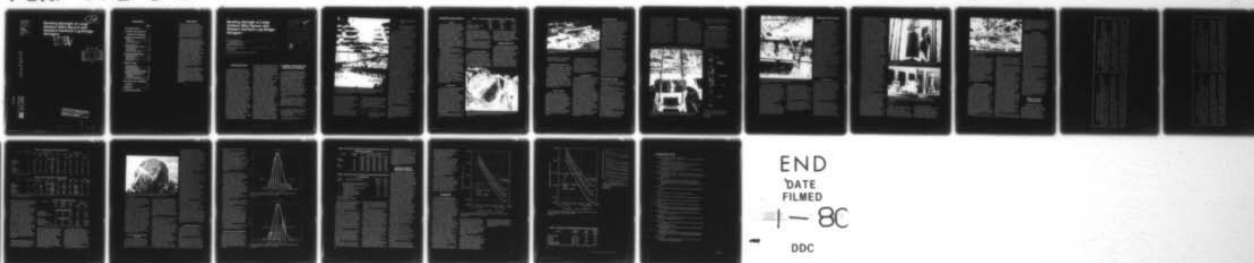
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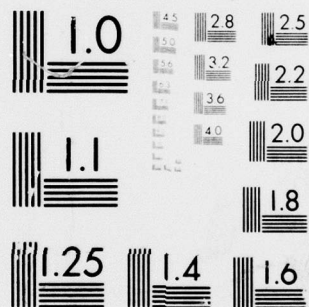
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**Bending Strength of Large
Alaskan Sitka Spruce and
Western Hemlock Log Bridge
Stringers**

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ABSTRACT

Native log stringer bridges are important economical and practical structures for logging roads in remote areas of Alaska. Unfortunately, the current knowledge of the strength of large logs is extremely limited. In the absence of actual test data, design stresses have been estimated by procedures developed for poles and piles.

To obtain actual data on the strength of large logs, a field test facility was designed and 40 large logs were tested to destruction. These were probably the largest logs ever tested, with butt diameters up to 4 feet, 10 inches and ultimate bending loads in excess of 120,000 pounds. This study was limited to two species, Sitka spruce and western hemlock. The average breaking strengths were 4,530 pounds per square inch (lb/in.²) for Sitka spruce and 4,680 lb/in.² for western hemlock. Both values are reasonably close to those obtained by the current design procedure. Statistical curves for estimating near-minimum strengths at various confidence levels were developed.

It is normal practice in the design of wood structures to reduce the allowable stresses to a 10-year duration of maximum load. This is unrealistic for bridges where the maximum design load is present for only a short period. A cumulative load duration of 2 months is more realistic. A simple curve for stress adjustments at different load periods is presented.

Bending Strength of Large Alaskan Sitka Spruce and Western Hemlock Log Bridge Stringers!

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INTRODUCTION

Native log stringer bridges are important and practical structures on both public and private forest lands. They are built from trees cut in the proximity of the bridge site in remote areas where it is not economically feasible or practical to bring in the more conventional construction materials. Several hundred bridges of this type are currently in service in Alaska and Canada, and the more remote areas of the lower 48 States. They carry heavy off-highway logging trucks with loads exceeding 100 tons in spans approaching 100 feet (fig. 1).

Federal regulations now require that all bridges on Federal and Federal-aid roads be inspected and load-rated by recognized engineering procedures. The bridges are then posted for the maximum vehicle weights permitted to use the bridges. In a number of instances, the calculated allowable load for log stringer bridges was much less than the weight of logging trucks which had been regularly using these bridges for several years. It was apparent that new reliable design information was needed to properly analyze log stringer bridges.

Unfortunately, the current knowledge of the strength of large logs is extremely limited. Considerable

information is available on the strength of small clear wood specimens, and methods have been devised to account for strength-reducing characteristics such as knots and slope of grain with sawn wood products. These limiting defects for structural grades are readily discernible in lumber and are described in the grading rules. No such rules exist for full-sized logs.

Factors that are assumed to affect log strength include: knots, spiral grain, growth eccentricity when associated with reaction wood, diameter, span, density, growth rate, taper, and stiffness. Unfortunately, few of these characteristics are simple to measure on large logs in the field and little research has been conducted to assess the influence of these nine variables. In the absence of test data, the allowable design stresses for bending are based on procedures developed for poles and piles (2)¹.

Three factors that control the structural safety of log bridges are: The initial bending strength of the logs; the deterioration in strength following years of service; and the magnitude and distribution of the loads. Whereas companion studies are underway on the latter two factors, this report is limited to the first. To determine their ultimate strength, a total of 40 large logs, 25 Sitka spruce, and 15 western hemlock, were field-tested in bending.

CURRENT METHODS OF DERIVING STRESSES

Log stringer bridges are designed by two different methods. One assumes relatively low fiber stresses, but allows full or nearly complete load distribution between stringers. Gower suggests fiber stresses of 1,350 lb/in.² for Douglas-fir and 950 lb/in.² for spruce, and assumes that 5/6 of the total number of stringers are available to resist live load (6,7). Pratt uses 1,200 lb/in.² bending stress for Coast Douglas-fir (14), while Klima suggests 1,300 lb/in.² for Douglas-fir and 1,100 lb/in.² for western hemlock (9). Both consider the live-load moment to be divided equally by the number of stringers.

The U.S. Forest Service (USFS) in Alaska, Alaska Region, permits higher allowable stresses, but imposes a more severe distribution factor, whereby it is assumed that 0.3 of the live-load moment is carried by a single stringer

¹A cooperative study between Alaska Region and the Forest Products Laboratory of the U.S. Forest Service, the Ketchikan Pulp Company, Ketchikan, Alaska, and the Alaskan Lumber and Pulp Company, Sitka, Alaska.

²Maintained at Madison, Wis., in cooperation with the University of Wisconsin.

³Underlined numbers in parentheses refer to literature cited at end of this report.

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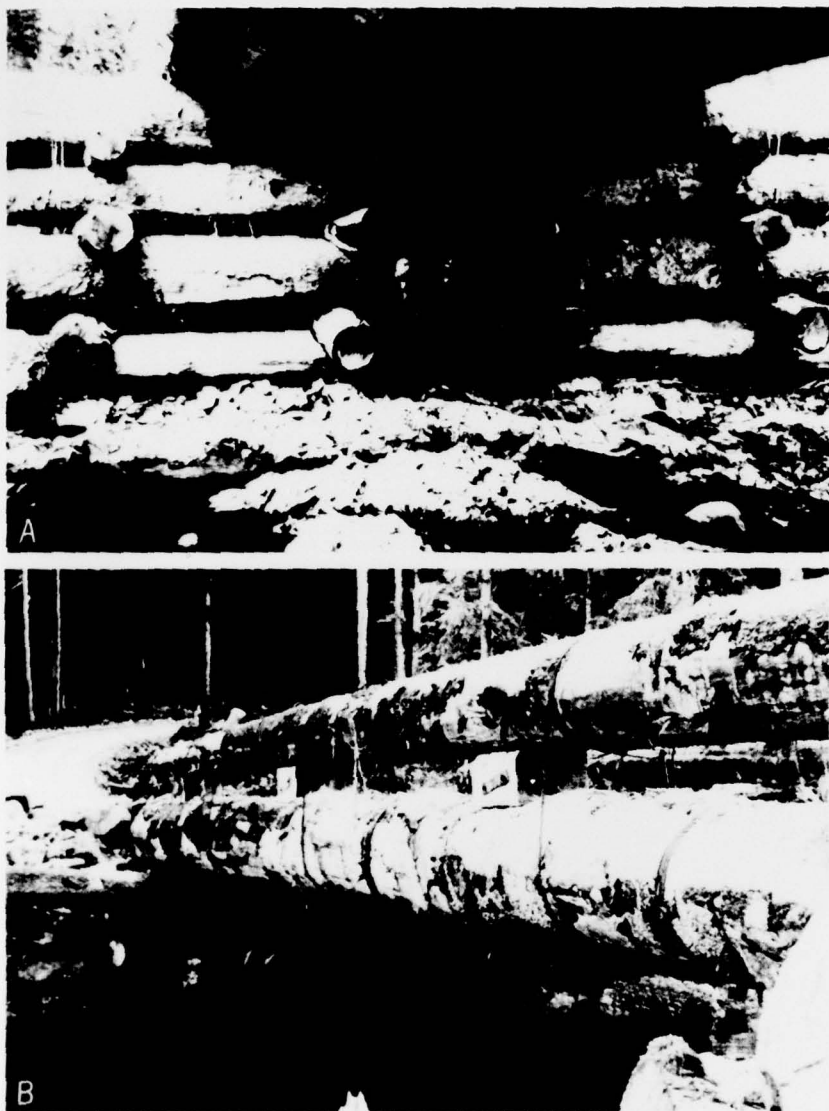


Figure 1.—Log stringer bridges. A—large log stringers are simply placed on log crib abutement. B—bridge with spread brow logs; this bridge spans nearly 100 feet and is designed for 80-ton off highway logging trucks. (M 143 536) (M 143 751-16)

(12). The reason for the differences between the Alaska Region and others is probably in construction detail. The Region uses a greater number of stringers with a rock fill running surface. The other designers use fewer stringers with a timber deck. Some recent designs use truss cables to increase performance (8).

The log stringer design stresses in the USFS procedure are based upon ASTM D2899. Standard Method for Establishing Design Stresses for Round

Timber Piles (2). This procedure results in higher stresses than those proposed by Gower, Klima, and Pratt, but was adopted for log stringers because of the unavailability of any other procedure. Design stresses in bending by the ASTM method are calculated as follows:

$$f_b = \frac{(S - 1.645 SD)}{2.04} \quad (1)$$

where f_b = working stress for extreme fiber in bending for green untreated timber.

S = average small clear bending strength from ASTM D2555.

and

SD = standard deviation of small clear bending strength from ASTM D2555.

The 2.04 is an adjustment factor which includes: Adjustment from short-time test conditions to normal duration of load; variation in strength along the length; adjustments for variability; and a factor for growth and shape characteristics. The working stresses are for green untreated construction poles and piles. The strength data needed for the above equation for most North American species are contained in ASTM D2555 (3).

The USFS follows this procedure and further reduces the allowable stresses obtained above by a safety factor of 1.3 as recommended in paragraph 13.1 of ASTM D2899 (2). It should be noted that the first adjustment (1.645 SD) is to obtain a statistical point estimate at which 95 percent of population should exceed that strength level. The Alaska Region used this procedure, except that the strength properties for Sitka spruce and western hemlock were based on samples of wood from Alaska (15). This was done to account for any differences indigenous to trees grown in that geographical area. The allowable bending stresses for three species, as derived by ASTM procedure are given in table 1 (12).

The lifetime, or inventory rating, is the load level that the bridge can safely support continuously for a period of at least 10 years. The short-term rating is for bridges that will be in service for only 1 to 2 years. Generally they are removed from service after a single logging operation. A 15 percent increase in allowable stress is currently permitted for these bridges based upon a cumulative load duration of 2 months. The one-time overload (operating rating) is the maximum permissible load level to which the bridge can be subjected for a short duration. It allows a 33 percent increase above the lifetime rating, assuming a cumulative load duration of 1 day.

Sitka spruce and western hemlock are the species predominantly used in southeast Alaska and this study is limited to these two. Douglas-fir values are presented in table 1 simply for comparison with the design stresses used by others as previously discussed.

RESEARCH MATERIALS

Source

Most log stringer bridges in Alaska are constructed with Sitka spruce or with western hemlock. For the results of this study to be of maximum usefulness, logs were selected from three widely separated locations within the 16-million-acre Tongass National Forest to get a representative sample. From a practical standpoint, the logs had to be obtained from existing logging operations. No land transportation links connect the three sources of test logs considered representative of these two species in southeastern Alaska. Both Sitka spruce and western hemlock logs were obtained from:

- False Island (Chichagof Island)—midway between Sitka and Juneau;
- Zarembo Island—between Ketchikan and Juneau; and
- Prince of Wales Island—about 60 air miles northwest of Ketchikan.

The test site was located on Prince of Wales Island and specimens from that area were trucked to the site. Logs from the other two sites were towed or rafted to the test area. Two major pulp companies—Alaska Lumber and Pulp Company of Sitka and Ketchikan Pulp Company—cooperated with the Forest Service by furnishing the stringer logs from their logging operations. Each log was identified as to its source.

Due to logistics, no samples were obtained from the very northern part of the Tongass National Forest—the Yakutat Area. However, the number of log stringer bridges in the Yakutat Area is relatively small.

Sample Size

The number of replications necessary to establish reasonable confidence of near-minimum strength was first determined by nonparametric techniques (4). With this method, no calculations are required. The lowest observed value of the sample is taken as the desired tolerance limit. Confidence levels depend on the sample size. The method assumes nothing is known about the distribution pattern and probably yields conservative results. During the planning stage, it is highly useful in setting sample size, which obviously must be a compromise between reasonable confidence and the cost of testing.

For this study, the lower 10 percent exclusion limit was selected as the near-

Table 1.—Allowable bending stress for three species of logs

Rating	Sitka spruce	Western hemlock	Douglas-fir
	Lb/in. ²	Lb/in. ²	Lb/in. ²
Lifetime of bridge (8-10 yr)	1,600	1,800	2,000
Short term (1-2 yr)	1,800	2,000	2,300
One-time overload	2,100	2,400	2,600

minimum value to be estimated. Confidence levels of 80 percent were desired for western hemlock and 90 percent for Sitka spruce. The higher confidence was desired for Sitka spruce because it is used in a greater number of bridges.

The required sample sizes for these two species were 15 western hemlock logs and 22 Sitka spruce. However, extra Sitka spruce logs were delivered to the site, and 25 were actually tested.

Log Size and Quality

Samples were selected on the basis of Forest Service standards (11) that require high quality, straight, sound logs that are free of shake and decay. Logs must have a slope of grain less than 1 in 8, no knots exceeding 5 inches in diameter in the middle half, and no two knots directly opposite one another.

These logs were probably the largest

ever tested destructively. The largest log had a butt diameter of 4 feet 10 inches (fig. 2). Midspan diameters ranged from 19 to 41 inches with lengths of 45 to 75 feet. All logs were cut within a few weeks of actual testing. The logs were not debarked.

TEST FACILITY

It was impractical to ship such large logs to a testing laboratory for full-scale evaluation, so a field test facility was built on Prince of Wales Island at Thorne Bay, a yarding center. The initial estimate of maximum test loads was about 100,000 pounds, and the test procedure and equipment were designed for this load. The total equipment shipped to the site weighed about 4,000 pounds. The test facility consisted of a sorting and inspection area, an anchor system, a loading system, and a data acquisition system.



Figure 2.—Record-sized logs were tested to destruction in the field. This large Sitka spruce log had a butt diameter of 4 feet, 10 inches. (M 143 751-12)



Figure 3.—Overview of the test site. The support cribs were shifted along the base rails for span adjustment. (M 143 750-7)

The site was within a rock quarry that provided a good base, ample space, and easy access from the road.

Sorting and Inspection Area

A sorting and inspection area was set up adjacent to the test area. It consisted of pairs of log rails that supported the test logs above ground prior to testing. The logs were first sorted according to length so that logs of similar span were accessible for sequential testing.

The bottoms of the test logs were approximately 3 feet above the ground. A space of similar dimension was left between adjacent logs so that all sides were accessible for inspection. Lumber stops were nailed to the rails to prevent the logs from rolling.

Anchor System

A positive anchor system was essential to provide a safe reaction point, and was designed by Alaska Region engineers. Twelve vertical rock bolts and anchors were first drilled and expanded into the rock substrata to define a 10- by 10- by 15-foot rock mass. Additional rock bolts were angled into the foundation rock and anchored the mass to a wide-flange steel beam straddled by three steel channel sections. All anchor bolts were torqued to the required levels with a calibrated torque wrench to ensure design capacity.

A 200,000-pound-capacity anchor bolt was then attached directly to the steel beam with three 1- by 12- by 12-inch steel base plates, four nuts and bolts, and the anchor bolt nut. The anchor bolt was made of 2-inch-diameter round bar and had an inside eye dimension of about 8 inches. The load linkage was attached directly to the eye of the anchor bolt.

Loading System

The loading system was comprised of three subsystems: The support cribs; the connecting linkage; and the power source. The support cribs had to be adjustable to accommodate various spans, and also be free enough to simulate simple supports. The connecting linkage had to transmit imposed test loads between the log, anchor point, and the power source without creating stress concentrations on the test specimens. Additionally, the load linkage had to provide an adequate mechanical advantage to overcome the difference between specimen strength and available power source. The power drive from field sources had to be flexible. Three loading mechanisms were considered: The direct pull from a crawler tractor; the power take-off from a crawler tractor; and a stationary drum hoist. Without the mechanical advantage of sheave blocks, none had the capacity to supply the necessary force directly for the anticipated ultimate loads.

Supporting Cribs

The supporting cribs consisted of two pairs of base rails laid parallel to the length of the test specimen (fig. 3). Five additional large logs were cross stacked above each pair of rails. The top log was hewed level as a base for the rocker supports. The total height of about 18 feet was required for the load cell, sheave blocks, and connections.

Rocker supports were positioned on top of each crib (fig. 4). Outriggers were attached to each rocker support to increase lateral stability. Glued-laminated wood saddles were attached to each rocker. To provide a smooth contact surface the saddles were machined to radii of 22 inches for the butt support and 16 inches for the tip support.

A second smaller crib was built around the anchor bolt to serve as a work platform and also protect the load cell and equipment during testing.

An auxiliary mast was built and attached to the protection crib. It had a small manual hoist for raising and lowering the sheave blocks to make connections to the test logs. The mast cable was always left attached to the top sheave block to prevent it from falling to the ground after a test failure. Sufficient slack was provided so that there was no interference with the load application. Safety chains were provided for the butt rocker to prevent the log from falling during and after testing. The tip was free to slide in the other support.

Connecting Linkage

The connecting linkage is shown schematically in figure 5. It consisted of a girth strap, shackles, sheave blocks, and a load line.

The girth strap was needed to distribute the load uniformly to the top surface of the logs to prevent compression failures. It was made up of a number of 6- by 18-inch metal plates. Two round brackets were welded to each plate, and spacer sleeves were installed between adjacent plates. Cables were then threaded through the sleeves and brackets. The sleeves provided a slight gap between plates so that the assembly was flexible enough to match the curved surface of the log. Thimbles and shackles were attached to each end for direct connection to the upper sheave block.

Two quadruple sheave blocks (fig. 6) were inserted between the girth straps at the top and the load cell at the bottom. The sheaves were only 12

inches in diameter, the minimum size for blocks of 50-ton capacity. Special $\frac{3}{4}$ -inch 6 by 37 wire rope was needed to provide the necessary flexibility for use with such small sheaves.

The sheave blocks were reeved for eight-part line. (It was imperative that the proper reeving instructions were strictly followed to keep the loads balanced and avoid damage to the equipment.) This provided a mechanical force advantage of close to 8:1. It also reduced the speed of loading by the same ratio, which was of equal importance. The load line was extended horizontally from the bottom sheave directly to the power source.

Power Source

The selection of power source was controlled by the availability of equipment from the logging operations. Testing started with a crawler tractor with a draw bar capacity of about 30,000 pounds but this produced too irregular rate of loading and the power takeoff drum had a rather limited cable capacity. Thus, a double-drum yarding hoist was located for the test program.

The yarding hoist was mounted on skids, and was oriented perpendicular to the final line of force so that it could be positively anchored. A swivel block at the front end provided directional

flexibility. The hoist had ample power, was extremely smooth in operation, and provided good starting and stopping control.

Data Acquisition System

The applied forces were measured with a commercial load cell with a rated capacity of 100,000 pounds (with a capable short-time capacity of another 100,000 lb). It was calibrated at Forest Products Laboratory (FPL) (fig. 7) to a maximum of 150,000 pounds, both before and after the field tests. No changes in calibration were noted. A second load cell was made and calibrated at FPL to serve as a backup cell in case the commercial one became damaged during testing, but was not needed.

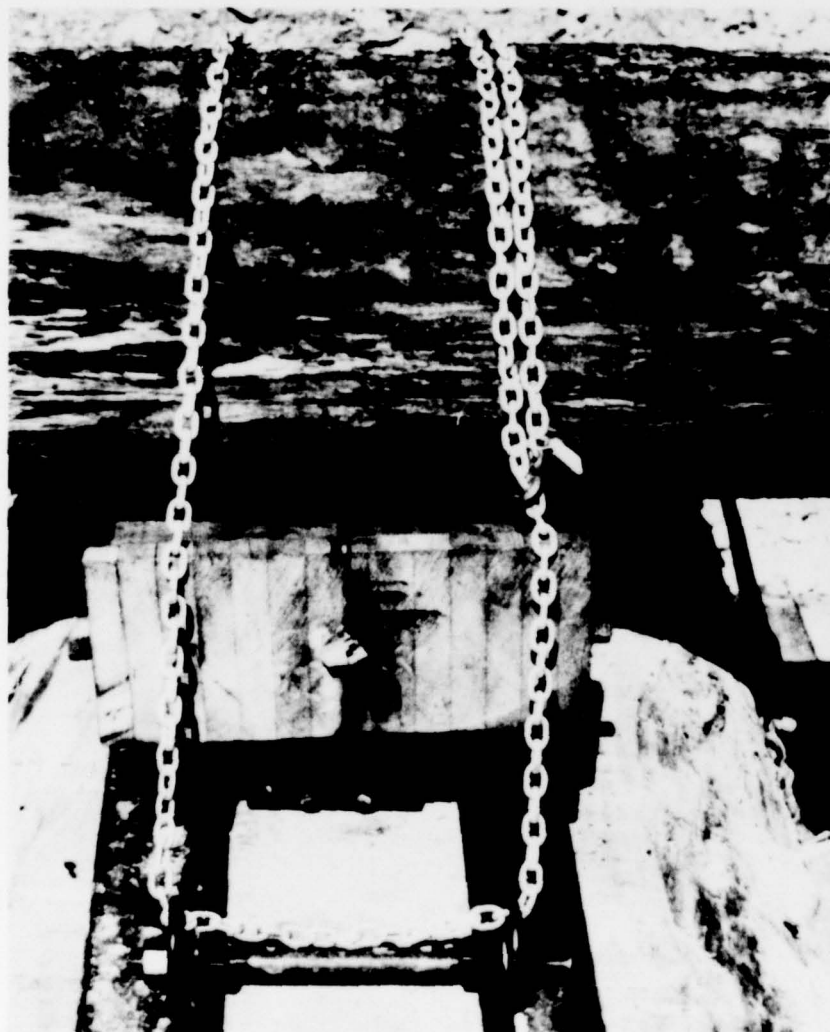


Figure 4.—Rocker support with log butt resting in saddle. The butt end was chained down for safety and the tip was free to slide at the other end. (M 143 750-19)

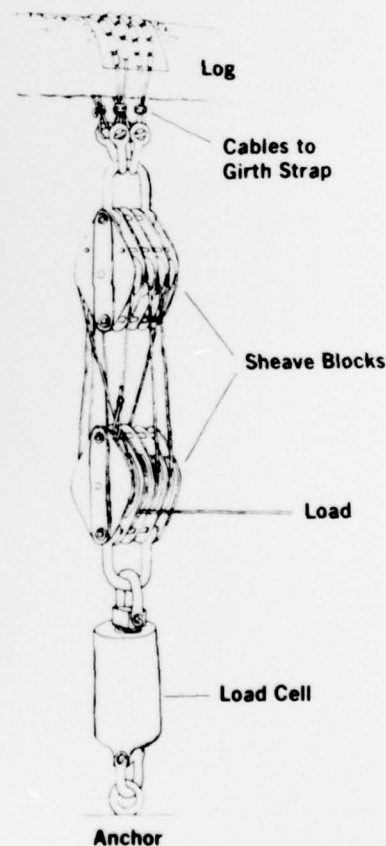


Figure 5.—Schematic of the loading system. Two quadruple sheave blocks were used to obtain a mechanical advantage. (M 143 980)



Figure 6.—Girth strap and upper sheave block. The girth strap was designed to reduce compressive stress on the top of the log.
(M 143 750-16)

The load cell was inserted between the bottom sheave block and the ground anchor, and measured the net vertical force at the reaction point. Load readings were monitored continuously with a portable digital strain indicator. It was necessary to manually follow and balance the gage circuit to obtain the strain readings. The instrumentation cables were housed in pipe between the load cell and strain indicator for protection from the weather and mechanical damage.

Deflection readings were taken optically with a transit. A level rod (scale) was mounted on each test log at midspan. A constant tension reference wire was stretched between large nails driven into the log near the neutral axis directly over each support. The wire

was tied off at the butt, and a ring bearing was used at the tip. A deadweight hung from the wire at the tip so that the tension in the wire remained constant. Then, by adjusting the horizontal cross hair to coincide with the reference wire, net deflection readings could be obtained without the need to measure support deflections or compressive deformation at the log ends.

All of the measurements were taken at a distance of about 60 feet from a control shelter, which was the body of an old crew bus (fig. 8). The distance was sufficient so that parallax and angle changes were insignificant.

RESEARCH METHODS

Visual Examination

Prior to testing, each log was given an identification number, inspected for various physical characteristics, and marked for orientation at the time of inspection (fig. 9). The identification number included origin, species, and an inspection sequence number. Characteristic data recorded included knot information, slope of grain, circumference at 5-foot intervals, and comments on eccentricity of the pith due to reaction wood. The mapping orientation mark gave a means of accounting for the effect of knots under a different test orientation.

Knot size and location were recorded over the interval extending 10 feet from either end of each log. Knot locations were recorded as the longitudinal distance from the butt end with the corresponding circumferential coordinate measured from an arbitrary reference line at the top of the log. Two orthogonal measurements taken along the maximum and minimum diameters of the knot were used to record its size. The slope-of-grain measurements were taken over a length of 10 feet close to the center of the log.

In order to determine the degree of taper, measurements of circumference outside bark were taken at 5-foot intervals from tip to butt. A linear regression equation was then derived for each log describing the taper and intermittent diameters.

Test Procedure

The initial thought was to test the logs as cantilevers to reduce the required applied force (1). However, for that type of loading the ground reaction points would have to be considerably stronger, and they would have been less flexible with respect to span changes. Also, from a safety standpoint, cantilever loading would be more dangerous because mechanical connectors would have been required at the free end of the log. For these reasons, it was decided to test the logs as simple beams.

Two orthogonal baselines were established through the center of the anchor bolt. One baseline was parallel to the axis of the test log and was used for centering the rocker supports. The other baseline was used for centering the support cribs and squaring the rocker supports. The door of the control shelter was centered on this line, so that

readings could be taken from inside. Two permanent reference points were established on each baseline.

The support cribs were centered optically. Each rocker support had two reference pins, one on each side above the rotational axis. The rockers were first centered in the direction of the log axis. A tape was then stretched between the two reference pins, on the same side of the two rockers, and the support cribs were slid along the rails until they were both approximately equidistant from the anchor bolt. The rocker supports were then shifted on the flat surface of the top crib log until they were within ± 0.1 foot of the desired span. They were then squared up by rotating them until the distances from the two reference points on each block to midspan were equal. The rockers were then secured with cable clamps. Logs were not tested in any specific order; rather those of near equal length were tested consecutively to minimize the number of span changes.

A large log stacker (fig. 10) was used to move the test logs. The heavy girth strap was laid over the log near midspan prior to placing the log on the supports. Once the log was in place, the butt end was chained down for safety. The girth strap was next centered at midspan.

The upper sheave block was next raised using the mast hoist, and was connected to the girth strap with four shackles. The hoist line was slacked off but left connected to the top sheave block to catch it following failure.

The reference wire was then strung between the two ends of the log near the neutral axis. A roller bearing was inserted over the tip connector and a deadweight was hung from the wire to provide constant tension. A scale was mounted on the log at midspan approximately 1 inch from the wire. Finally, the lead line from the bottom sheave was connected to the load line from the yarding hoist.

Loads were controlled by deflection in increments of approximately 0.1 foot (fig. 11). The instrumentman had visual contact with the hoist operator and gave hand signals to start and stop loading. A zero load cell reading was taken prior to loading, and then load readings were taken at each deflection increment. Each 0.1-foot increment required about 10 seconds of loading time, and the pause for readings required about another 10 seconds. The dial on the strain indicator was turned continuously to balance the electrical circuit and measure the load at each

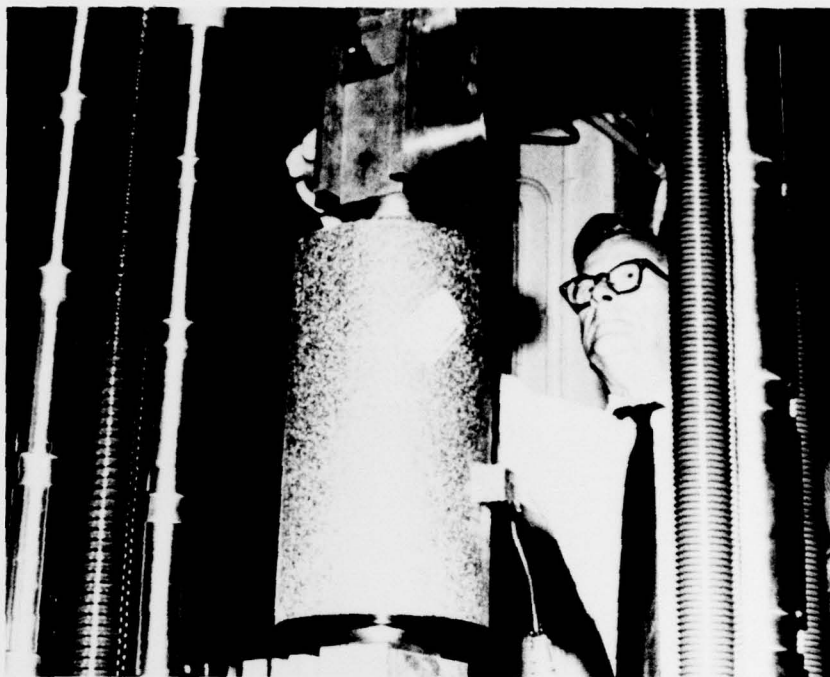


Figure 7.—A commercial 50-ton capacity load cell was calibrated before and after field tests.
(M 143 751-5)



Figure 8.—Test data was monitored from this control shelter. Deflection readings were taken optically by transit and strain readings with a digital voltmeter.
(M 143 750-12)

increment.

The time to failure depended on the number of load increments required to cause failure. The number of increments ranged from 5 to 19, and averaged about 12. At 20 seconds per increment, the average failure occurred in about 4 minutes, which is reasonably

close to laboratory testing.

After test, each log was removed from the supports and its failure zone inspected. Increment core samples were taken at midspan and shipped to FPL for analysis of growth rate and specific gravity. The failure zone was also inspected for knots and other

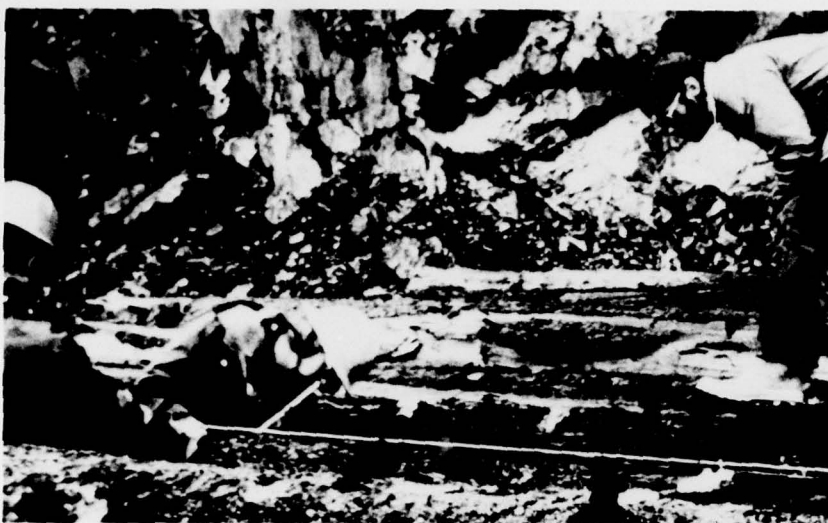


Figure 9.—Mapping defects and obtaining physical measurements on 40 test logs in the sorting area.
(M 143 750-8)

possible causes of failure.

Neither the hoist operator nor the log stacker were assigned full time to the testing project. Following a test, all of the load devices were disconnected. The stacker was then summoned to remove the broken log and place a new log on the supports. This took about 5 minutes and the stacker was released for its normal work. After all the test preparation was completed, the hoist operator was called in for about 10 minutes to load the beams.

With this procedure and a crew of four, five to eight logs could be tested in 1 day, depending upon the number of span changes required.

Methods of Analysis

The modulus of rupture (MOR) was calculated as the ratio of the maximum moment to the section modulus at the critical location. In standard laboratory testing of small clear specimens, the deadweight of the specimens can be neglected. However, with logs of massive cross sections, the deadweight stresses are significant, and were included based upon an assumed unit weight of 40 lb/ft³. The weight of the load equipment above the load cell (slightly over 1,200 lb), although less significant than the deadweight of the log, was also included in the stress analysis. The increase in stresses due to deadweight ranged from 6 to 14 percent for western hemlock and 6 to 23 percent for Sitka spruce.

The modulus of elasticity (MOE) was

calculated from the linear portion of the load-deflection curve. The equation for deflection of prismatic circular beams was used with an adjustment factor to account for taper. Shear deflection was neglected.

Both MOR and MOE were based upon diameters measured outside the bark. These values would have been higher had the section modulus and moment of inertia been adjusted to inside bark dimensions; the impact of such adjustments is considered later in report. However, for design and rating purposes field measurements include the bark, and thus strength values must be developed on the same basis.

An attempt was made to adjust the section modulus to account for knots, similar to the strength ratio principle for solid timber and the I_k/I_g concept for glulam. Knots were considered to be truncated cones with the base at the surface and the apex at the pith.

A procedure was developed to adjust the section modulus based on the knot map data. First, the knot locations were oriented to correspond with the position of the log during each test. Then, net section moduli were calculated in small increments along the log length. Stresses were calculated from the moment diagram using the net section properties along the log at each knot location. This refinement yielded stresses that ranged up to 30 percent higher than those based upon the full section.

However, these areas of higher stress were confined in narrow bands and did

not generally coincide with the fracture zone. The effect of radial knots in a circular section is considerably different than knots in a rectangular section, and no meaningful correlations could be found between the knot properties and the MOR's.

Calculations

The calculations of strength and stiffness for round tapered beams is more complex than for prismatic sections, and no handbook formulae were found. It was necessary to derive equations specifically for this study.

The analysis of round beams actually has three solutions depending upon the span, diameter, and taper. The first solution is the prismatic case, namely zero taper. The second case is where the butt diameter is more than 1.5 times the tip diameter, in which case the maximum stress occurs away from the load point. The third solution applies to cases where the butt diameter is less than 1.5 times the tip diameter. In this case the maximum stress occurs at the load point when located at midspan or at some point between midspan and the tip end of the log. All test logs were of this third type.

To minimize the required load, the theoretical load point for all logs would have been between midspan and the tip support. But this would have required adjustment of the crib supports for every test. Because ample capacity was available, all tests were applied at midspan for expediency.

Since the derivations of the equations for strength and stiffness of round tapered beams are rather lengthy and beyond the scope of this paper, they are not included in this report. Also, the solutions have broader application than this one study, because logs are used for poles, posts, beams, joists, rafters, and purlins. Therefore, the equations and derivations will be published in another report.

RESULTS AND DISCUSSION

Description of Failures

All logs failed near midspan. They exhibited two distinct modes of failure, the normal and the abrupt modes. The normal mode produced a load-deflection curve displaying a linear elastic region, a fairly distinct yield point, and finally a plastic region. In the abrupt mode, failures occurred while

U.S. Forest Products Laboratory.

Bending Strength of Large Alaskan Sitka Spruce and Western Hemlock Log Bridge Stringers, by R. L. Tuomi, R. W. Wolfe, R. C. Moody, and F. W. Muchmore. Madison, Wis., For. Prod. Lab., 1979.

18 p. (USDA For. Serv. Res. Pap. FPL 341).

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the load-deflection curve was still fairly linear or immediately after the yield point was reached.

The normal failures were quite gradual, starting from shallow and elongated cracks on the tension side. Logs exhibiting normal failures usually stayed up on the supports and retained a certain amount of residual strength (fig. 12). The abrupt type failed suddenly, often falling off the supports (fig. 13).

The western hemlock logs, although slightly stronger than Sitka spruce, failed more often in the abrupt mode. The failure zone had a brash appearance. Seven of the 15 failed abruptly, with five breaking completely and falling from the supports. Only five of the 25 Sitka spruce failed abruptly and only one fell from the supports.

The weakest Sitka spruce ($MOR = 3,020 \text{ lb./in.}^2$) failed without warning when the load-deflection curve was still linear. There were no unusual characteristics in the log to explain the sudden failure or low strength. It was the only spruce log that fell from the supports. The two weakest western hemlock ($MOR = 3,180$ and $3,700 \text{ lb./in.}^2$), failed in a similar manner, also with no discernible defects. All three logs were in the shortest class, tested over a 40-foot span, and failed at deflections of only 6 to 7 inches. The average deflections at failure for the normal failures was about 12 inches for the same span. It is possible that these three logs were damaged during the felling operation and had preexisting compression failures not detected when the logs were inspected.

Logs exhibiting abrupt failures failed earlier, usually between the seventh to ninth load increment. Logs that failed in the normal mode, on the average, reached the 13th load increment.

Deflections were less than anticipated for green logs. Western hemlock logs were all tested on either 40- or 47-foot spans and the mean deflections at failure were 12 and 14 inches, respectively. Sitka spruce over the same spans deflected slightly more, 13 and 17 inches.

Sitka spruce was also tested at spans of 52, 59, 66, and 69 feet, but these cannot be compared directly with western hemlock. However, the overall deflection/span ratios at failure for the two species were not much different: 0.024 for hemlock and 0.025 for spruce.

Strength and Stiffness

The log strength and stiffness

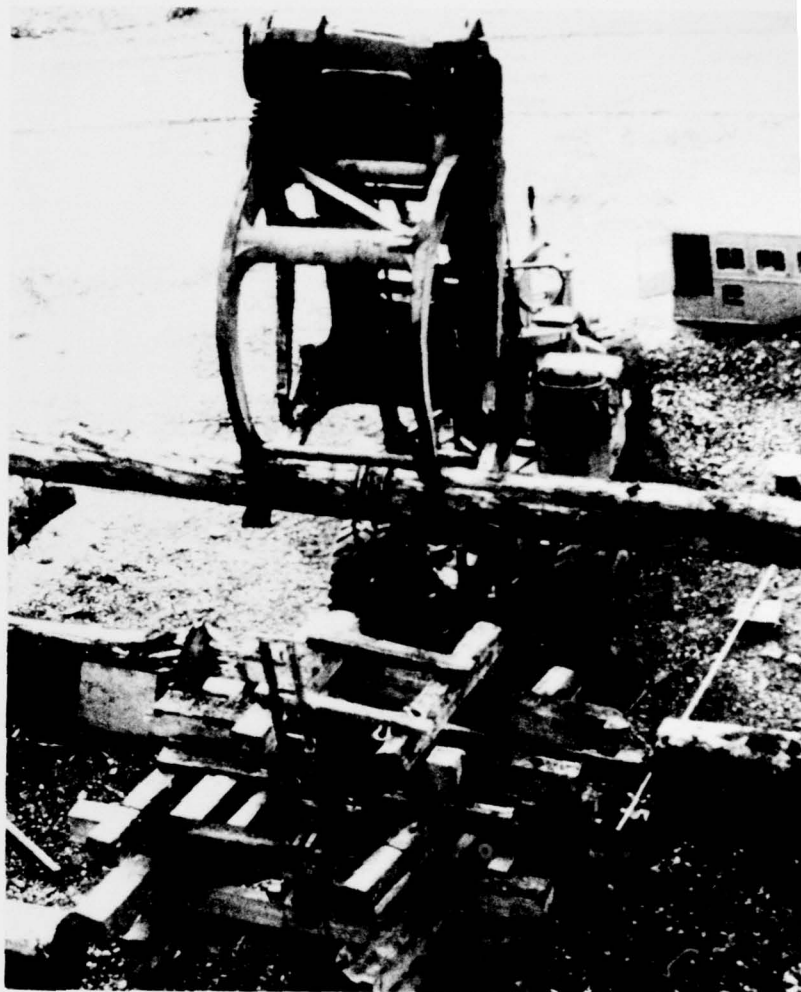


Figure 10.—Giant log stacker placing test log up on supports. Logs had to be lifted approximately 20 feet.
(M 143 750-13)

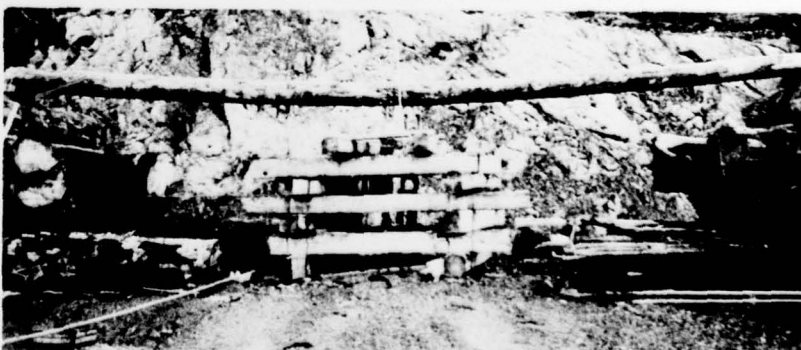


Figure 11.—Log under load over a 70-foot span. Loads exceeding 120,000 pounds were required to break the strongest logs.
(M 143 750-4)



Figure 12.—Sitka spruce logs failed gradually and usually stayed up on supports.
(M 143 750-10)

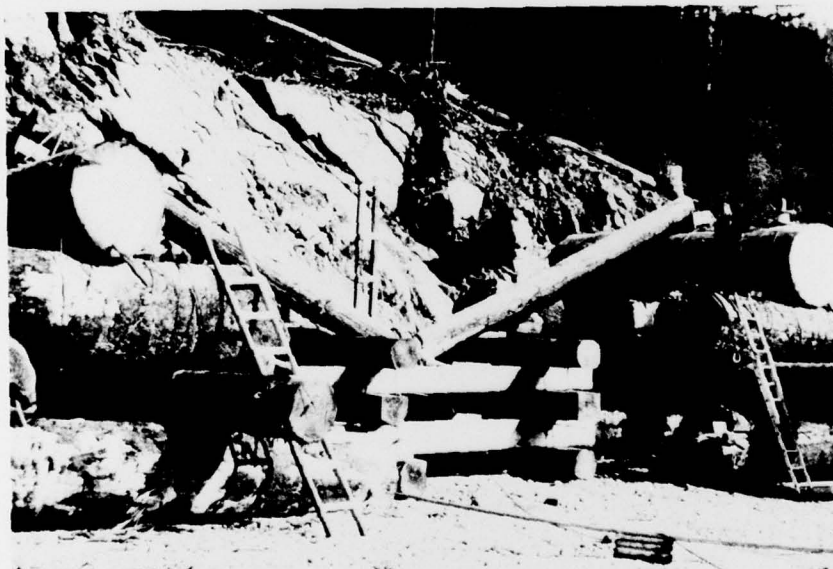


Figure 13.—Western hemlock logs, although slightly stronger than spruce, failed abruptly, often falling to the ground.
(M 143 750-5)

properties are presented in table 2. Sample estimates are subject to variation depending on inherent variability of the population and the size of the sample. At the 95 percent confidence limits, the mean MOR for the true western hemlock population should

^aAverage bark thickness of specimens from large trees on file at the Forest Products Laboratory.

fall between 4,330 and 5,200 lb/in.², and the standard deviation between 490 and 1,080 lb/in.². The corresponding 95 percent confidence limits for the true Sitka spruce population are 4,310 to 4,620 lb/in.² and 570 to 1,020 lb/in.².

The mean estimated strength of the western hemlock was 3 percent higher than that for Sitka spruce. This is

considerably less than the 12 percent difference reported for small clear wood specimens (3, 16). Also, in developing design stresses, the abrupt nature of the failures in western hemlock logs must be considered. Appraising both the strengths obtained and the method of failure, it appears questionable that the Alaskan western hemlock warrants design levels higher than the Sitka spruce.

There was little difference in average MOR based on point of origin or source. Both the strongest and weakest specimens for the two species came from Prince of Wales Island, and this material also produced the highest coefficient of variation.

The average MOE's 1.06 million lb/in.² for western hemlock and 1.23 million lb/in.² for Sitka spruce, are somewhat lower than the average values reported for small clear specimens of the two species. However, as previously mentioned, MOE's were calculated on outside bark dimensions and would therefore be lower than values determined from actual net sections.

The overall coefficients of variation for MOR (16 pct) and MOE (22 pct) agree quite well with those values for clear wood (3).

Consideration of Bark Thickness

The above MOR and MOE values are based on diameters that include bark, but round timber stresses for poles and piles are used with diameters that do not include bark. By considering the bark to average 3/16 inch thick on the Sitka spruce and 9/16 inch thick on the western hemlock,^a new MOR and MOE values can be calculated that may correspond more closely to stresses for such round timbers. The MOE is more sensitive to bark thickness (a function of diameter to the 4th power) than is MOR (3rd power). Also, because the Sitka spruce logs averaged larger in diameter and had thinner bark, MOR and MOE values for Sitka spruce are less sensitive to the adjustments than those for the western hemlock.

The calculated mean MOR for western hemlock is 5,440 lb/in.² and for Sitka spruce 4,760 lb/in.² using estimated inside bark diameters. These values are 16 percent and 5 percent higher than the respective values using outside bark diameters. But the coefficients of variation remained essentially unchanged at 17 percent for both species. The average MOR of 5,440 lb/in.² for western hemlock is 11

Table 2.—Log strength and stiffness properties¹

Source	Number of logs	Modulus of rupture					Modulus of elasticity				
		Average	Standard deviation	Coefficient of variation	Maximum	Minimum	Average	Standard deviation	Coefficient of variation	Maximum	Minimum
		—Lb./in. ² —		Pct	Lb./in. ²	Lb./in. ²	MM lb./in. ²		Pct	—MM lb./in. ² —	
WESTERN HEMLOCK											
False Island	4	4,800	694	14.4	5,710	4,060	1.17	0.366	31.4	1.66	0.84
Zarembo Island	4	4,620	644	13.9	5,500	4,010	1.16	.203	17.6	1.33	0.92
Prince of Wales Island	7	4,660	987	21.2	5,830	3,180	0.94	.133	14.1	1.11	0.79
All hemlock	15	4,680	784	16.7	5,830	3,180	1.06	.241	22.8	1.66	0.79
SITKA SPRUCE											
False Island	8	4,540	543	12.0	5,660	4,080	1.04	.249	23.9	1.49	0.74
Zarembo Island	6	4,330	591	13.6	5,070	3,430	1.12	.104	9.3	1.30	1.01
Prince of Wales Island	11	4,630	1,053	22.8	6,870	3,020	1.42	.399	28.0	2.34	0.87
All spruce	25	4,530	797	17.6	6,870	3,020	1.23	.344	28.0	2.34	0.74

¹Based upon outside bark diameters.

Table 3.—Log stringer physical characteristics¹

Source	Number of logs	Growth rate		Taper ²		Midspan diameter ³		Slope of grain		Specific gravity	
		Average	Range	Average	Range	Average	Range	Average	Range	Average	Coefficient of variation
		Rings/in.		—In./ft.—		In.	In.	—In./in.—			Pct
WESTERN HEMLOCK											
False Island	4	17	14-20	0.14	0.11-0.19	22.5	18.9-25.3	0.089	0.042-0.167	0.39	10
Zarembo Island	4	18	10-34	.13	0.08-0.15	22.4	20.3-24.8	.052	0.001-0.125	.31	9
Prince of Wales Island	7	30	18-48	.15	0.11-0.18	23.5	20.5-26.5	.077	0.010-0.167	.45	12
All hemlock	15	23	10-48	.14	0.08-0.19	23.2	18.9-26.5	.073	0.001-0.167	.42	12
SITKA SPRUCE											
False Island	8	15	8-32	.13	0.09-0.16	28.9	25.1-33.6	.046	0.021-0.087	.39	13
Zarembo Island	6	8	7-11	.17	0.12-0.24	30.5	23.0-41.1	.026	0.007-0.050	.37	7
Prince of Wales Island	11	26	9-38	.14	0.09-0.20	29.2	20.0-34.1	.103	0.010-0.292	.40	9
All spruce	25	18	7-38	.14	0.09-0.24	29.4	20.1-41.1	.058	0.007-0.292	.39	10

¹Growth rate and specific gravity based on measurements made on a single increment core taken from each log near midspan following failure. Specific gravity was based on oven-dry weight and volume in the green condition. Taper, diameter, and slope of grain were measured in the field prior to test.

²Taper is the change in diameter per foot of length.

³Based upon outside bark measurements.

percent lower than the pole test results of McGowan (11), which averaged 6,135 lb./in.² for fifty-two 25-foot-long poles from British Columbia. Such a difference could be due to the difference in size: logs in this study were about three times as large as the poles.

The calculated mean MOE values using inside bark dimensions is 1.30 million lb./in.² for both the western hemlock and the Sitka spruce. The coefficient of variation in MOE remained essentially unchanged at 24 and 28 percent for the two species. McGowan (11) found an MOE of 1.66 million lb./in.² in his pole tests of western hemlock, which is considerably higher than the 1.30 million lb./in.² value for the logs. However, the mean values of 1.30 million lb./in.² for both species are identical to results found for green, clear wood specimens from southeast Alaska (15), and within about 5 percent of values published by ASTM for U.S. species (3).

Physical Characteristics of Logs

The physical characteristics of the logs are summarized in tables 3 and 4. As mentioned earlier, the factors

Table 4.—Log stringer knot data					
Source	Number of logs	Average number of knots per log	Mean knot area ¹	Total knot area per log	
				Average	Range
			ln. ²	ln. ²	ln. ²
WESTERN HEMLOCK					
False Island	4	13	11.6	187	0-414
Zarembo Island	4	32	15.3	391	193-713
Prince of Wales Island	7	3	11.6	37	3-79
All hemlock	15	16	12.8	171	0-713
SITKA SPRUCE					
False Island	8	26	12.6	345	75-909
Zarembo Island	6	60	11.8	768	32-1,203
Prince of Wales Island	11	19	11.9	166	0-510
All spruce	25	35	12.1	368	0-1,203

¹The knot area was calculated from the average of the major and minor knot diameters.

assumed to affect log strength were: Knots, spiral grain, growth eccentricity when associated with reaction wood, diameter, span, density, growth rate, taper, and stiffness. However, analyses of MOR and each of these nine variables failed to produce any meaningful correlations. Apparently these logs were similar in quality and these limited data were too closely grouped to establish definite trends. Also information on knots was limited to surface measurements.

Slope of grain (spiral grain), for example, could only be measured on the surface. Spiral grain is not constant throughout the bole, but changes during the growth of a tree from right-hand to left-hand spiral. With lumber, slope of grain generally does not follow this pattern and its effect on strength has been determined. But with circular sections and changing spiral grain, the effects are not so obvious.

The knot density data are summarized in table 4. As mentioned earlier, an



Figure 14.—Cross section of a failed log showing knot growth and shake. Only the knot in upper left quadrant surfaced while three others did not. (M 143 750-15)

unsuccessful attempt was made to modify the sections to account for the knot areas. The effect of knots on the moments of inertia of circular sections is not as direct as assumed for rectangular sections.

Figure 14 shows the cross section of a log taken through a surface knot near the zone of failure. The major knot appears at the surface in the upper left quadrant and essentially approximates the shape used in the calculations for knot moment of inertia. However, there is evidence of three additional knots, one in each quadrant, that did not appear on the surface. Consequently, the influence of knots would be difficult to assess since only the surface knots are visible. Although it is logical to specify logs with a minimum number and size of knots, their effect on strength could not be defined in large members in this study.

Estimates of Near-Minimum Strengths

A tolerance limit is an estimate of a near-minimum property that a given percent of the population will exceed. For example, a tolerance limit of 90 percent with a MOR of 3,000 lb/in.² implies that, on the average, 9 out of 10

random logs in any one bridge will possess bending strength in excess of 3,000 lb/in.². If a higher tolerance limit is required, such as 95 percent, then this strength level must be reduced so that, on the average, not more than 1 log in 20 falls below that near-minimum strength. Estimates of the lower fifth percentile (95 pct tolerance limit) are implied as a basis for design in much of the literature on structural properties of timber.

Some confidence level is associated with a tolerance limit and it reflects the degree of certainty (or uncertainty) inherent in the estimate. The uncertainty arises from the fact that only a sample of the total population was (or is ever) tested. Test results give data for the sample while properties are needed for the population. Confidence levels are selected on professional judgment and safety needs, but have not generally been formally applied to engineering design. European and American standards organizations are presently considering confidence levels near 75 percent for use in multiple member timber structures.

At a 90 percent tolerance limit, the nonparametric technique produced a minimum MOR of 3,180 lb/in.² with a 79

percent confidence for western hemlock, and 3,020 lb/in.² with 92 percent confidence for Sitka spruce. In some cases, species can be combined where the properties are similar as is done with the southern pine groups, for example. Combining both the western hemlock and Sitka spruce logs yields a minimum estimate for MOR of 3,020 lb/in.² at a 90 percent tolerance with a confidence of 98 percent. The nonparametric technique used here determines the statistical significance of the weakest observed specimen, but provides no information on the distribution pattern.

Frequency distributions for the two species are presented in figures 15 and 16. Both the normal and lognormal distributions are superimposed over the histogram. Experience with lumber and glulam testing suggests that the lognormal method often better represents the population distribution. Lognormal distributions eliminate zero and negative strength which physically do not exist, and techniques are available for estimating near-minimum values. Near-minimum strengths for various confidence levels as estimated by lognormal distributions are presented in table 5. Values were developed from coefficients published by Natrella (13).

The common use of a formal factor of safety is not compatible with the tolerance / confidence approach. However, current design stresses can be evaluated by considering this factor to be 1.3 as given in table 6.

As shown in figures 17 and 18, current designs are at about the 99 percent tolerance limit for Sitka spruce, and 97 percent for western hemlock at the 0.90 confidence level. The associated MOR does not take into account load sharing, whereby the stronger and stiffer logs assume a greater portion of the applied load.

Current Design Stresses

To determine how the current method of deriving design stresses (2) compares with actual field tests, it is necessary to examine the procedure step by step. The breakdown for deriving design stresses is contained in table 6.

Step 1 simply records the average basic small clear wood strengths that have been established by testing (3, 16). It should be noted that the values used by the Alaskan Region are based on samples from only five or six trees (15), but the MOR's were within 4 percent of

the population averages from ASTM D2555 (3).

Step 2 contains a combination of factors to account for several factors such as variability, growth characteristics, form (shape), and size. All of these variables were inherent in the logs tested, and Step 2 should relate directly to the average MOR's. Compared to the average log strengths in table 2, the current procedure yields results that are only 3 percent high for Sitka spruce and 12 percent high for western hemlock—quite close agreement.

Step 3 is a statistical approach to estimate the near-minimum strength at the fifth percentile. It is analogous to the 95 percent tolerance limit at the lower side of the frequency distribution. However, it assumes a normal distribution of the population and does not have an associated confidence.

A safety factor (or load factor) of 1.3 is applied in Step 4 to obtain short-term design stresses. This is a deterministic design process to resolve uncertainties and provide for some margin of safety. From a statistical or reliability standpoint, this adjustment factor reduces the near-minimum values with a corresponding increase in confidence.

Step 5 is an adjustment for duration of load which assumes that the bridge is under full design load continuously for a 10-year duration.

Figures 17 and 18 indicate how the present design stresses using lognormal distribution relate to various tolerance limits at four different confidence levels. Prior to the 10-year duration of load adjustment, the short-term design stress for Sitka spruce at the 99 percent tolerance limit carries a 90 percent confidence level. Western hemlock at the same tolerance would have a confidence level slightly less than 75 percent.

As mentioned earlier, design stresses for western hemlock are less conservative than for Sitka spruce.

Duration of Load Consideration

Step 5 in table 6 is the normal duration of load adjustment to convert the short-term strengths to a 10-year loading. Experience has shown that the capacity of timber decreases in an exponential (hyperbolic) manner as the duration of load increases, and the current practice is to assume that the full design load is present for a period of 10 years.

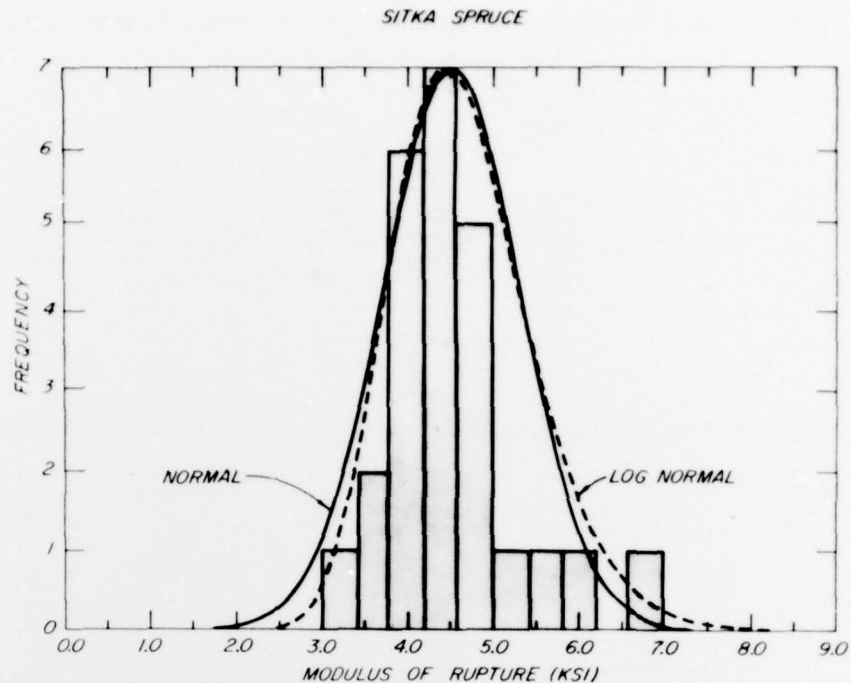


Figure 15.—Histogram of the strength of Sitka spruce logs showing both normal and lognormal frequency distribution. (M 146 421)

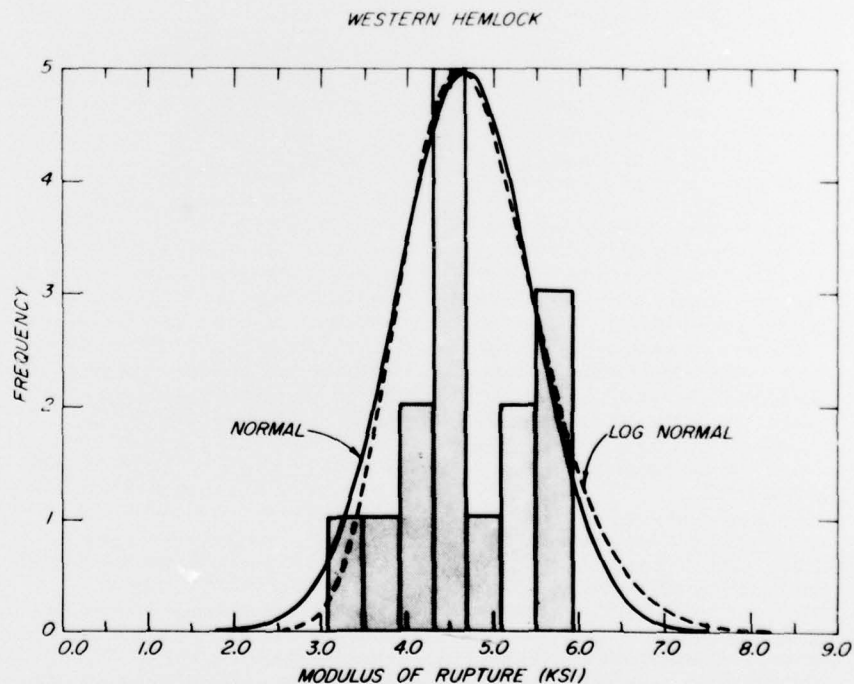


Figure 16.—Histogram of the strength of western hemlock logs showing both normal and lognormal frequency distribution. (M 146 420)

Table 5.—Near-minimum modulus of rupture estimates using lognormal distribution assumptions

Species	Confidence level	Near-minimum modulus of rupture for one-sided tolerance limits				
		75 pct	90 pct	95 pct	99 pct	99.9 pct
Western hemlock	75	Lb/in. ² 3,980	Lb/in. ² 3,560	Lb/in. ² 3,320	Lb/in. ² 2,910	Lb/in. ² 2,510
	90	3,840	3,390	3,140	2,710	2,290
	95	3,740	3,280	3,020	2,570	2,150
	99	3,540	3,040	2,760	2,290	1,850
Sitka spruce	75	3,850	3,440	3,200	2,810	2,420
	90	3,750	3,310	3,080	2,660	2,270
	95	3,680	3,240	2,990	2,570	2,160
	99	3,540	3,080	2,820	2,380	1,960

Table 6.—Present procedure for determining allowable bending stresses for log stringers

Step number	Strength adjustment phase	Western hemlock	Sitka spruce
1	Basic small clear wood strength, MOR ¹	Lb/in. ² 36,600	Lb/in. ² 25,900
2	Basic adjustments for log form and size (divide by 1.26)	5,240	4,680
3	Point estimate or the 5th percentile (multiply by 0.737)	3,860	3,450
4	Short-term design stress (divide by 1.3 safety factor)	2,970	2,650
5	Adjust to 10-year loading (divide by 1.62)	1,830	1,640

¹From Res. Pap. FPL 1 (15).

²Based on samples taken from 5 trees.

³Based on samples taken from 6 trees.

Some recent studies suggest that the cumulative 10-year loading might be unrealistic (5, 10). Very few wood structures are subjected to the maximum load for the life of the structure. This would be particularly true of bridges.

For example, a truck traveling at 15 miles per hour across a 60-foot bridge would be on the bridge for less than 3 seconds. Assuming a traffic volume of 10 trucks per hour, for 12 hours per day, and 6 days per week during a 6-month logging operation, the cumulative loading per year would be less than 16 hours. And during the 3 seconds that a truck is on the bridge it is only at the position of maximum load for a fraction of that time.

More efficient designs might result if engineers could accurately predict the actual cumulative load durations. A more realistic estimate of cumulative loads on a logging bridge would be about 2 months. This would allow an increase in design stresses of about 15 percent.

The Ministry of Transportation and Communication, Province of Ontario, Canada, has taken an approach similar

to this in their Ontario Bridge Design Code. They have proposed a 2-month duration of load for bridges, which is longer than anticipated during 30 years of service.

The one-time overload (operating rating) covers those periods of maximum load when heavy logging equipment, yarding towers, and loaders cross the bridge. This loading occurs only from two to six times per year. At 1 minute per crossing, this equates to a cumulative duration of 6 minutes per year or 1 hour during the lifetime of the bridge. One week should be an ample duration for a short-term bridge.

Duration of load curves are generally presented as semilog curves and are rather difficult to interpolate accurately for different units of time. Simplified duration of load curves are given in figure 19 that provide direct adjustment factors for different time units.

The curves were derived from an empirical hyperbolic equation based on work conducted by Wood (17). Coefficients for the empirical equations were modified to fit different time units, so that the stress ratios could be represented by a family of curves with a

common abscissa. For example, to find the adjustment for a 6-month cumulative load enter "6" on the X-axis, move vertically to the "month" curve, and read 1.10 on the Y-axis. For a "6" hour adjustment, continue upward to the top (hour) curve, and read 1.38 off the ordinate. These factors should be multiplied by the conventional 10-year design load commonly published for wood products.

EVALUATION OF PRESENT DESIGN

It is impossible to design log stringers that will never fail. No structures, not even major ones, can realistically be designed with zero probability of failure. Experience with log stringer bridges has shown that an occasional stringer does break, but the failure of a single stringer has not been known to cause the bridge to collapse. Ninety-three percent of the bridges in the Alaskan Region have seven or more log stringers, and many have supported loads until the broken stringer was replaced. This situation would not hold for bridges constructed with only two, three, or four stringers.

Evaluation of the present design for Sitka spruce logs using the results of this is presented in table 7. Western hemlock values are not included because it is doubtful that it justifies higher design stresses than Sitka spruce. The procedure used is similar to that used in the glulam industry, where design values may be developed from test data rather than from small clear wood values. The basic stress is based on the 95 percent tolerance limit and 75 percent confidence. This value is reduced by dividing by 1.3 to account for test variables and uncertainties. It is again divided by 1.62 to obtain 10-year load stresses. From table 5, the corresponding MOR (3,200) for Sitka spruce reduces to a 10-year design stress of 1,530 lb/in.².

As discussed in the preceding section, bridges are not under full design load for long durations. It is estimated that 2 months, 1 week, and 1 hour are reasonable durations for the lifetime, short-term, and one-time overload ratings. Increases in allowable stresses appropriate for these durations were obtained from figure 19 and compared with the present design stresses.

The present design stresses are all lower than the results obtained by this study. Load durations are cumulative, but not for all three ratings. Lifetime and

one-time overload are cumulative, and short-term plus a portion of the one-time overload are cumulative. The present design stresses provide adequate reserve to cover the cumulative durations.

Inspection records should provide a reasonable indication of how well the current stress levels are working. The number of observed stringer failures can be compared to the expected values presented in table 5 or figure 17. If traffic demands change, requiring more conservative design values, new allowable stresses can be derived for the appropriate tolerance and confidence levels from the data contained herein.

For rating purposes, it might be possible to use different stresses depending upon the hazard involved. For example, a short-span bridge over a shallow depression poses less danger than a long bridge over a deep ravine. Higher design stresses could be justified in the first case. Such an approach would require some system of potential hazard classification.

SUMMARY

A test facility was designed that proved capable of testing extremely large logs in the field. Test procedures were also developed that closely parallel laboratory testing. Forces of up to 120,000 pounds were applied with reasonable control of the load rate. Although field tests can never equal the accuracy attainable in a laboratory, they can be successfully conducted within acceptable limits. These were probably the largest logs ever tested destructively and provide basic knowledge on the strength of stringer-sized logs.

The current design procedure is based on theoretical extrapolation of data on much smaller specimens. The average MOR's for the two species was within 3 percent for Sitka spruce and 12 percent for western hemlock of the values obtained from the current ASTM design procedure. The ASTM procedure appears to be reasonable in the absence of actual test data.

No meaningful correlations could be made between assumed strength-reducing characteristics and ultimate breaking strength. This is probably because those characteristics, such as knots discernible at the surface, were not necessarily indicative of the entire cross section of the log.

Statistical methods were employed to estimate the near-minimum bending

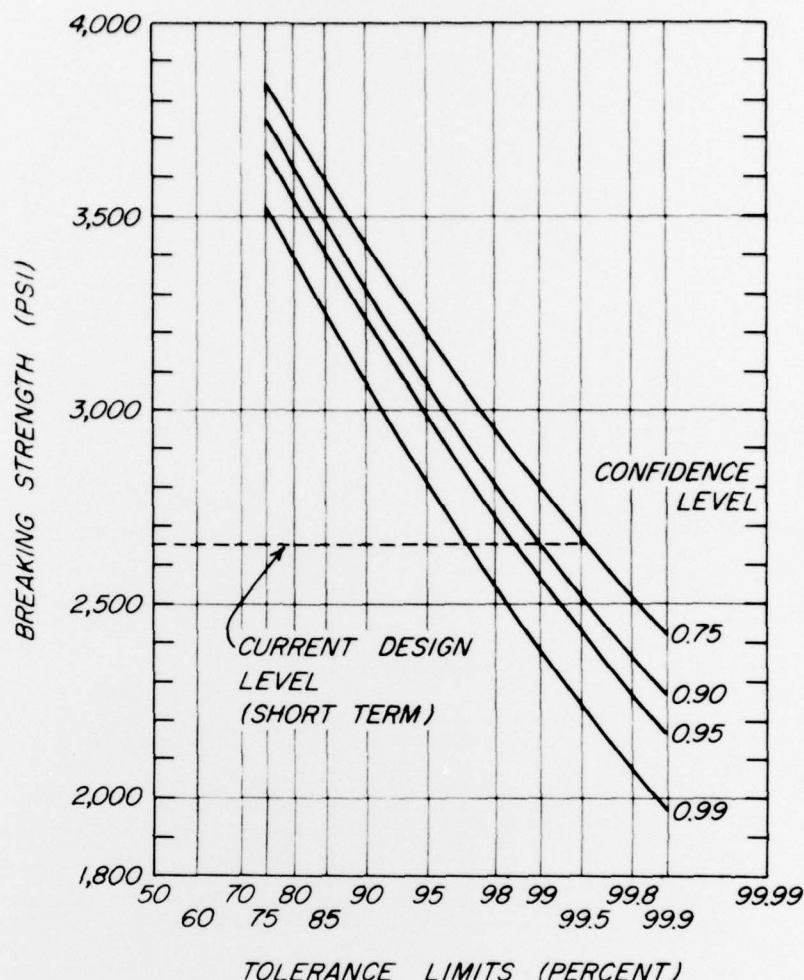


Figure 17.—Estimate of the percent of Sitka spruce logs which will exceed a given strength for four confidence levels. (M 146 418)

strength of the logs. The lognormal technique, which has been used for solid lumber and glulam, was used to develop strength distributions. The normal safety factor reduces the minimum strength with a corresponding increase in confidence. Curves were produced for the two species so that a range of minimum strengths can be selected for different levels of confidence.

The 10-year duration of load factor, which generally applied to wood structures, is unrealistic for log stringer bridges. A cumulative load period of 2 months would be far more appropriate for lifetime duration, particularly if records of logging operations are available for load rating existing bridges. Curves were developed that

depict increases in allowable stresses for the anticipated time of loading on log stringer bridges. Present design stresses are lower than those derived from this study.

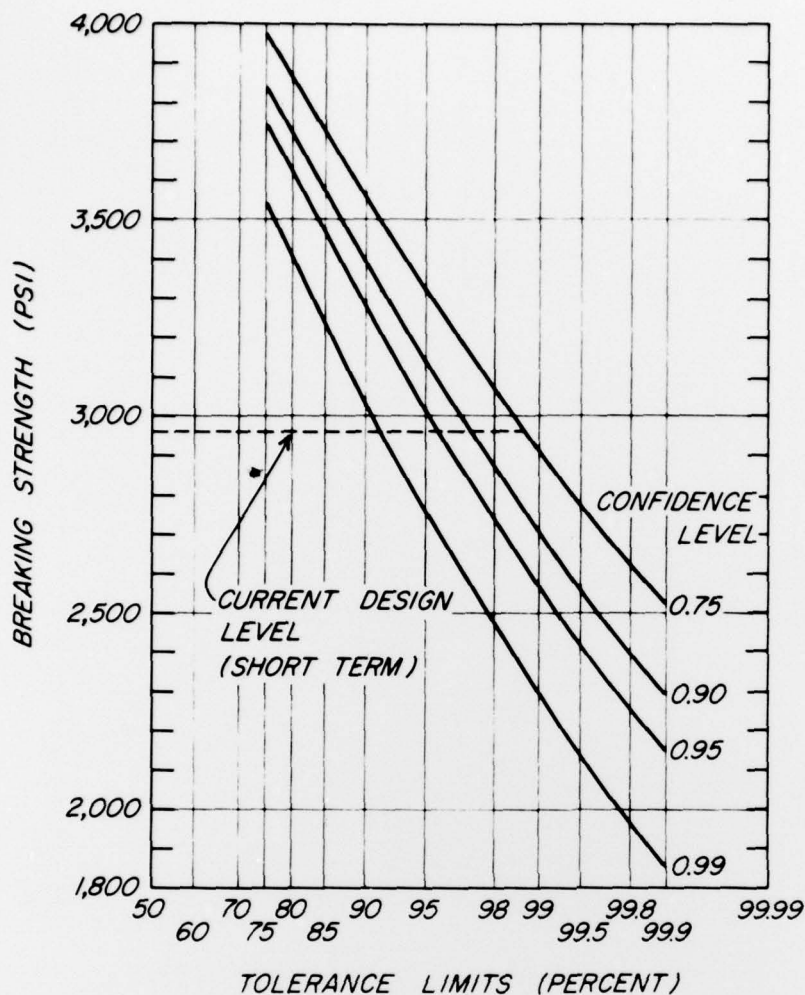


Figure 18.—Estimate of the percent of western hemlock logs which will exceed a given strength for four confidence levels.
(M 146 419)

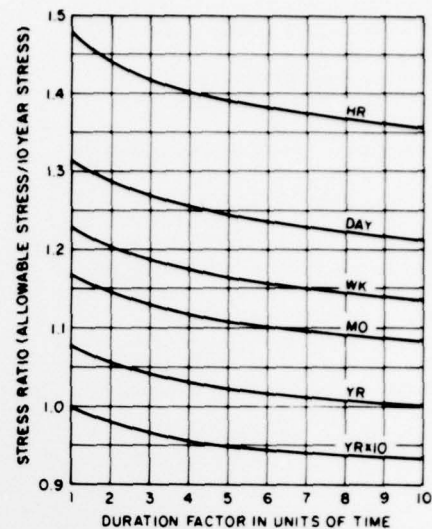


Figure 19.—Stress adjustments for cumulative durations of load for different units of time.
(M 146 416)

Table 7.—Evaluation of present bridge designs for Sitka spruce logs

Loading	Cumulative duration	Study results	Present design stress
		Lb/in. ²	Lb/in. ²
Lifetime of bridge (8-10 yr)	Use 2 mo; SF ¹ = 1.15	1,760	1,600
Short term (1-2 yr)	Use 1 wk; SF = 1.23	1,880	1,800
One-time overload (operating rating)	Use 1 hr; SF = 1.47	2,250	2,100

¹SF is stress factor or the increase in allowable stress for cumulative load durations less than 10 yr.

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The log strength and stiffness

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